C6.6 Piers

C6.6.1 General

C6.6.1.1. Policy overview

Methods Memo No. 181: Office Policy for Checking Piers by LRFD
1 December 2007

Some rather significant differences have been noted between piers designed using the LRFD Specification versus the Standard Specification. The differences have prompted the new policy below.

Piers that are approved to be designed by the LRFD Specifications shall be designed, as a minimum, by the design engineer using the LRFD code with RC-Pier. The check engineer shall check the design using the LRFD and the Standard Specifications. Check engineers shall use RC-Pier to do the LRFD pier check and the in-house pier programs to perform the Standard Specifications check. Checks done according to the Standard Specifications shall use HS20 live loading.

This policy is to help get a better understanding of the difference between the two specifications. Significant differences between the two checks with respect to pile/spread footing design, column reinforcement design, and cap reinforcement design should be brought to the attention of Mike Nop and Gary Novey.

This policy is effective immediately and will remain in effect until further notice.

C6.6.1.1.1. Frame pier

C6.6.1.1.2. Pile bent

C6.6.1.1.3. Diaphragm pier

C6.6.1.2. Design information

C6.6.1.3. Definitions

C6.6.1.4. Abbreviations and notation

C6.6.1.5. References

C6.6.2 Load application

C6.6.2.1. Dead

C6.6.2.2. Live

Methods Memo No. 40: Exterior Beam Distribution Factor -- LRFD
28 August 2001

See C5.4.2.2.2.

C6.6.2.3. Dynamic load allowance

C6.6.2.4. Centrifugal force

C6.6.2.5. Braking force

C6.6.2.6. Vehicular collision force

C6.6.2.7. Water

Methods Memo No. 145: Pier Foundation Design and Check for Scour Conditions
4 September 2007

See the Appendix for obsolete and superseded memos in this commentary.

C6.6.2.8. Wind

C6.6.2.8.1. Horizontal pressure on superstructure

C6.6.2.8.2. Horizontal pressure on substructure

Methods Memo No. 72: Application of Wind Load to Substructure
27 August 2002

For typical girder and slab bridges the AASHTO specifications permit use of simplified loads for wind on superstructure and wind on moving live load [AASHTO-I 3.15.2.1.3]. In the new release of the Office Design Manual (Article 6.6.2.6), the Office policy was changed to extend use of the simplified loads to span lengths of 140 feet (42.670 m). For span lengths greater than 140 feet (42.670 m) the designer shall use the more precise AASHTO loading [AASHTO-I 3.15.2.1.1]. This policy should be used on new projects that have not been designed.

C6.6.2.8.3. Vehicles on superstructure

C6.6.2.8.4. Vertical pressure on superstructure

C6.6.2.9. Ice

Methods Memo No. 82: Internal River Pier Ice Loads
12 January 2004 (References to the AASHTO standard specifications are obsolete.)

Since the present office policy was established, the Cold Regions Research and Engineering Laboratory (CRREL) has determined additional river pier ice loading information. Also, the AASHTO LRFD specifications make use of the latest information and differ somewhat from the AASHTO standard specifications. Consequently the office is updating policy as follows for Iowa internal river pier ice loads (excluding Mississippi River, Missouri River, and lakes).

Iowa internal river pier ice loads shall be determined from the following modified AASHTO standard specifications formula [3.18.2.2.1]:

$F = C_n C_b C_w ptw$

Where:

C_n = coefficient for nose inclination taken from the table in the AASHTO standard specifications [3.18.2.2.1].
C_b = coefficient for b/t (Pier Width/Ice Thickness) interpolated from the table in the AASHTO specifications [3.18.2.2.4].

C_w = reduction coefficient for bridges less than 300 feet (91 400 mm) long, from the AASHTO LRFD specifications commentary [C3.9.2.3]. The bridge length is a conservative assumption that was adopted by the office, so the designer would not have to estimate the stream width. C_w (K_1 in the LRFD specifications) shall be interpolated from the AASHTO LRFD specifications commentary [Table C3.9.2.3-1] based on A/r^2 (A = area of ice floe, r = radius of pier nose). Area of ice floe may be estimated as a circular area with diameter equal to the larger of two-thirds of the opening between the pier and abutment or between piers.

p = effective ice strength = 200 psi (1.38 MPa)

t = thickness of ice in contact with pier, inches (mm). Thickness shall be selected from the following table. These thicknesses were determined from the formula in AASHTO LRFD specifications commentary [C3.9.2.2]. The coefficient α was taken as an intermediate value for average and small rivers, 0.4, and the freezing index was taken as the 50-year (98%) value for a central location in the District, usually the District Office. Note the AASHTO LRFD coefficient of 0.4 is the same for both English and Metric units.

District	Thickness, inches (mm)
5	15 (380)
1, 4, 6	17 (430)
2, 3	19 (480)

w = width of pier stem or diameter of circular pier shaft at level of ice action, inches (mm). In cases where the pier is skewed to the flow, the projected width shall be used. The projected width will increase the ice load considerably, and if the load seems excessive the designer should investigate a circular pier shaft or other pier alternatives.

The following table compares the ice loads for a typical 3-foot-thick (910 mm) T-pier shaft with a vertical nose, at the center of the river, and aligned with the flow. The present load in the table includes the reduction effect of the coefficient associated with b/t in the AASHTO standard specifications [3.18.2.2.4].

District	Updated Load, Bridge Length >300 Feet (91 400 mm), kips (kN)	Updated Load, Bridge Length 100 feet (30 500 mm), kips (kN)	Present Load, River Any Width, kips (kN)
5	103.68 (460)	83.98 (370)	116.64 (520)
1, 4, 6	121.68 (540)	98.16 (440)	116.64 (520)
2, 3	139.54 (620)	113.03 (500)	116.64 (520)

Reference

Haynes, F.D. “Bridge Pier Design for Ice Forces,” *Ice Engineering Information Exchange Bulletin (Cold Regions Research and Engineering Laboratory),* No. 12, December 1995

The AASHTO LRFD specifications give no guidance for friction angle, θ_f. The paper by Montgomery et al., in which the equation that would become AASHTO Equation 3.9.2.4.1-1 was proposed, suggested 10 degrees. The more recent Canadian bridge code, however, suggests a more conservative 6 degrees, which was adopted for the manual.

Canadian Standards Association (CSA). [2000]. *CAN/CSA-S6-00, Canadian Highway Bridge Design Code,* Ottawa.

Montgomery, C.J., R. Gerard, W.J. Huiskamp, and R.W. Kornelsen. [1984]. "Application of Ice Engineering to Bridge Design Standards," *Proceedings: Cold Regions Engineering Specialty Conference, April 4-6, 1984, Montreal, Quebec*, pp 795-810.

C6.6.2.10. Earthquake

Summary, 4 June 2008: The AASHTO LRFD seismic requirements were made considerably more complex in the 2008 interim. The 2007 specifications varied the restrained horizontal connection design forces in Seismic Zone 1 as either 0.1 or 0.2 times the tributary load, but the 2008 interim sets the horizontal connection force as either 0.15 or 0.25 times the tributary load. The 2008 interim also removes an elastomeric bearing requirement that clouded the use of a friction coefficient of 0.2.

2007 AASHTO LRFD Specifications: From the beginning of the seismic article through the Seismic Zone 1 requirements there were 10 non-blank pages organized in what seemed to be a logical sequence.

- The design process depended on the Acceleration Coefficient, A. For Iowa, A varied between 0.02 for northeastern Iowa to 0.053 in southwestern Iowa [AASHTO-LRFD Figure 3.10.2-2].
- Because all Iowa Acceleration Coefficients did not exceed 0.09, all of Iowa was in Seismic Zone 1 [AASHTO-LRFD Table 3.10.4-1].
- Site Coefficients, S, varied from 1.0 for Soil Profile Type I (best profile) to 2.0 for IV (worst profile).
- For Seismic Zone 1 with $A \leq 0.025$ and Soil Profile Type I or II the horizontal design connection force in the restrained direction was 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 3.10.9.2].
- For all other cases in Seismic Zone 1 the horizontal design connection force in the restrained direction was 0.2 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 3.10.9.2].
- For elastomeric bearings a design coefficient of friction could be assumed as 0.2 between elastomer and clean steel or concrete [AASHTO-LRFD C14.8.3.1]. That assumption was clouded by another rule regarding friction at the strength limit state [AASHTO-LRFD 14.7.6.4] (which was removed in the 2008 Interim).

2008 AASHTO LRFD Specifications: From the beginning of the seismic article through the Seismic Zone 1 requirements there are 42 non-blank pages, many of which are maps.

- The seismic design process depends on three coefficients determined from maps and three factors determined from tables. Two coefficients and two factors are applicable to Seismic Zone 1 requirements.
- Seismic Zone 1 is defined by an acceleration coefficient, $S_{D1} < 0.15$, which is determined by $S_{D1} = F_v S_1$ [AASHTO-LRFD 2008 Table 3.10.6-1, 3.10.4.2]. From the map for Iowa, S_1 varies from 0.019 in northwestern Iowa to 0.044 in extreme southeastern Iowa [AASHTO-LRFD 2008 Figure 3.10.2.1-3]. F_v varies from 0.8 for Site Class A to 3.5 for Site Class E [AASHTO-LRFD 2008 Table 3.10.3.2-3]. Site Class is defined for different soil types and layers from A to F [AASHTO-LRFD 2008 Table 3.10.3.1-1]. Site Class F is defined for peat, very high plasticity clays, or more than 120 feet of soft/medium stiff clays. For Site Class F a site specific analysis is recommended.
- All of Iowa, with the possible exception of a few sites in extreme southeastern Iowa (Lee County) generally would be classified as Seismic Zone 1 as follows: northwestern Iowa $S_{D1} = (0.019)(3.5) = 0.0665$ maximum (except Site Class F); extreme southeastern Iowa $S_{D1} = (0.044)(3.5) = 0.0154$ maximum (except Site Class F). Extreme southeastern Iowa sites with Site Class E or F soil profiles could be considered Seismic Zone 2 under a strict interpretation of the AASHTO LRFD Specifications.
- The forces to be applied to bearing connections in Seismic Zone 1 depend on the short period acceleration coefficient, $A_s < 0.05$, which is determined by $A_s = F_{pga}PGA$ [AASHTO-LRFD 2008 3.10.9.2, 3.10.4.2]. From the map for Iowa, PGA varies from 0.015 in north central Iowa to 0.040 in southeastern Iowa. F_{pga} is to be taken for the applicable Site Class [AASHTO-LRFD 2008 Table 3.10.3.2-1].

- The A_s in Iowa will vary above and below 0.05 depending on Site Class. Therefore, the horizontal design connection force in the restrained direction will be either 0.15 or 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake [AASHTO-LRFD 2008 3.10.9.2]. In general the 0.15 factor will apply for northern Iowa and the 0.25 factor will apply in southern Iowa, but the designer will need to check for the specific bridge site.
- For elastomeric bearings a design coefficient of friction may be assumed as 0.2 between elastomer and clean steel or concrete [AASHTO-LRFD 2008 C14.8.3.1]. Generally if not restrained, steel reinforced elastomeric bearings in Seismic Zone 1 need not be anchored other than by friction [AASHTO-LRFD 2008 C3.10.9.2]. However, before making the final decision regarding anchorage, the designer shall consider the rules in the steel reinforced elastomeric pads article in the manual [BDM 5.7.4.2].

C6.6.2.11. Earth pressure

C6.6.2.12. Uniform temperature

C6.6.2.12.1. General

C6.6.2.12.2. Design temperature changes

C6.6.2.12.3. Out-of-plane forces

Policy discussion with Assistant Bridge Engineer
17 January 2003

There have been two guidelines in the office for determining the locations of axial fixity for piles. The older guideline is from the 1979 version of "Design Criteria for Piers":

> The pile length used to determine the footing rotation is assumed to be 50% and 75% of the pile length for timber and steel piling respectively.

The more recent guideline is the following:

> The percent of the pile length used to determine the footing rotation is assumed to be:
> (1) 50% when most of the pile capacity is due to friction bearing; generally wood, prestressed concrete, or steel piles not driven to bedrock, or
> (2) 75% when most of the pile capacity is due to end bearing, generally steel piles driven to bedrock.

For current design, use the more recent guideline. It is based on two concepts. The first concept is that a friction pile will transfer its load to the soil over the entire length of the pile, and thus axial deformation effectively will occur over half the pile length. The second concept is that an end bearing pile typically will transfer some load by friction and thus will not deform axially over its full length but over more length than a friction pile. The 75% value is a judgment factor.

C6.6.2.12.4. Unbalanced forces

C6.6.2.12.4.1. Bridges with integral abutments

C6.6.2.12.4.2. Bridges with stub abutments

C6.6.2.12.5. Skewed pier forces

C6.6.2.12.6. In-plane forces

C6.6.2.13. Temperature gradient

C6.6.2.14. Shrinkage

Summary 9 May 2008

Generally it is recognized that creep relieves a part of the stresses caused by shrinkage in restrained members but that it may be necessary to reinforce for shrinkage stresses. The AASHTO LRFD Specifications do not cover shrinkage in frames directly. However, for prestressed structures the specifications indicate that the designer need only consider initial elastic deformation for columns in monolithic frames and that other members will be subjected to reduced forces due to creep. Although there are no simple recommendations for shrinkage of frame piers, it is reasonable to use the AASHTO 28-day shrinkage coefficient of 0.0002 for design forces and provide the usual shrinkage and temperature reinforcement. Even though the shrinkage forces computed for 0.0002 will be reduced by the AASHTO LRFD load factor in strength limit state combinations, shrinkage forces will be included in every limit state rather than in only three load combinations under the AASHTO Standard Specifications.

Shrinkage research and standards

New York researchers (Antoni and Beal, 1971) instrumented a bridge pier to verify the AASHO 1964 specification requirements for coefficient of thermal expansion, temperature variation, and shrinkage. There were difficulties with the instrumentation, and the researchers could not separate temperature effects from shrinkage effects, but the researchers concluded that the 0.0002 shrinkage coefficient was reasonable.

In recent years there has been much study of concrete shrinkage in the hope that cracking could be reduced or eliminated. In overlays and composite structures the existing structure generally has undergone most or all of its shrinkage, and the new concrete is intended to be bonded to that structure and yet undergo shrinkage without cracking. Because of the infinite variety of concrete mixtures, the wide variety of structural conditions, the related factors such as thermal expansion/contraction and creep, and the differences between tension and compression creep, researchers have not found any simple and definitive answers to questions regarding shrinkage.

One researcher, however, has suggested that at two years creep reduces sustained tensile stresses by about one-third (Alexander 2005, 2007). The researcher also suggested that the designer provide reinforcement for tension caused by shrinkage.

The AASHO/AASHTO Standard Specifications have included the concrete shrinkage coefficient of 0.0002 since 1941. Specifications of that vintage had no commentary, so the source of the coefficient is unknown, except that New York researchers attribute the coefficient to experimental data from small laboratory specimens (Antoni and Beal, 1971).

The ISU strip seal report (Bolluyt et al. 2001) concluded that 0.0002 was an appropriate shrinkage value for concrete decks on prestressed beams or steel girders, although some of the bridges studied indicated more or less shrinkage. The report did not recommend any modification factors. (There was no proposal to use modification factors with thermal expansion/contraction either, but there was a recommendation to increase the temperature range for computing concrete bridge thermal movements.)

Of the surrounding six states, only Nebraska (0.0002) and Wisconsin (0.0003) include concrete shrinkage in their strip seal designs. In the ISU report there is no indication of whether the states include shrinkage in frame pier design.

The AASHTO LRFD equations for shrinkage and compressive creep [AASHTO-LRFD 5.4.2.3] show that both develop in the same pattern and, assuming the strain in concrete is about 10% of the crushing strain (among other assumptions), the creep and shrinkage curves approximately match. Using the shrinkage equation with assumptions, shrinkage can be computed of about 0.0007 at one year and 0.0008 at 2.5 years. The AASHTO LRFD

Specifications, however, generally indicate shrinkage of 0.0002 at 28 days and 0.0005 at one year [AASHTO-LRFD 5.4.2.3.1].

The AASHTO LRFD Specifications do require that effects of prestressing deformations on adjoining elements of the structure be evaluated [AASHTO-LRFD 5.9.2]. These deformations could be considered to be similar to shrinkage deformations. Additionally the specifications state the following:

> In monolithic frames, force effects in columns and piers resulting from prestressing the superstructure may be based on the initial elastic shortening.
>
> For conventional monolithic frames, any increase in column moments due to long-term creep shortening of the prestressed superstructure is considered to be offset by the concurrent relaxation of deformation moments in the columns due to creep in the column concrete.
>
> The reduction of restraining forces in other members of a structure that are caused by the prestress in a member may be taken as….

Considering the statements above, it is prudent to include some value of shrinkage in design. The statement regarding use of initial elastic shortening in monolithic frames could be taken to suggest the shrinkage coefficient of 0.0002 at 28 days. The next AASHTO statement could be taken to suggest that relaxation due to creep will relieve additional column moments due to shrinkage after 28 days. The AASHTO slowly imposed deformation equation (not copied above) with assumptions indicates that a 100-kip force, such as a cap force based on shrinkage, for time from 28 days to 896 days would reduce to 43 kips, less than half the initial force.

References

Alexander, S.J. (2007). "The Importance of Time in Understanding Concrete Behavior," *Conference on Structural Implications of Shrinkage and Creep of Concrete, SP-246*, pp 283-289.

Alexander, S.J. (2005). "Managing Deflection, Shortening and Cracking Arising from Restrained Contraction," *Shrinkage and Creep of Concrete, SP 227*, pp 1-20.

American Association of State Highway Officials (AASHO). (1941) *Standard Specifications for Highway Bridges, Third Edition,* American Association of State Highway Officials, Washington, DC.

Antoni, C.M. and D.B. Beal. (1971). *Temperature and Shrinkage Stresses in a Concrete Bridge Pier, Research Report 69-6*, Engineering Research and Development Bureau, New York Department of Transportation, Albany, New York.

Bolluyt, J.E., V.B. Kau, and L.F. Greimann. (2001). *Performance of Strip Seals in Iowa Bridges, Pilot Study, Final Report Project TR-437,* College of Engineering, Iowa State University, Ames, Iowa.

C6.6.2.15. Creep

C6.6.2.16. Locked-in force

C6.6.2.17. Settlement

C6.6.2.18. Friction

C6.6.2.18.1. Out-of-plane forces

C6.6.2.18.2. Unbalanced forces

C6.6.2.18.2.1. Bridges with integral abutments

C6.6.2.18.2.2. Bridges with stub abutments

C6.6.2.18.3. Skewed pier forces

C6.6.2.19. Vessel collision

C6.6.3 Load application to structure

C6.6.3.1. Load modifier

C6.6.3.2. Limit states

C6.6.3.3. Longitudinal and transverse forces transmitted through bearings

C6.6.3.3.1. Elastomeric bearings

C6.6.3.3.2. Fixed bearings and keyed-in concrete diaphragms

C6.6.3.3.3. Friction-acting bearings

C6.6.3.4. Longitudinal and transverse forces for non-skewed piers

C6.6.3.5. Parallel and perpendicular forces for skewed piers

C6.6.4 Pier Components and details

Methods Memo No. 181: Office Policy for Checking Piers by LRFD
1 December 2007

See C6.6.1.1.

C6.6.4.1. Frame piers and T-piers

When modeling frame piers and T-piers in structural software, the designer may assume a column to be fixed 2 feet (600 mm) below bottom of column if the column is supported on a pile footing or spread footing. In some software the single location of fixity will simplify design of the column and its foundation. The designer also has the alternative of using a spring connection representing footing rotation if a column requires more detailed analysis.

Methods Memo No. 16: Use of Higher Strength Concrete
21 March 2001

In situations where the design depths of pier caps and footings are beyond limits suggested because of longer bridge spans or special situations, consideration can be given to increasing the concrete strength for the pier concrete. Consider using structural concrete with a minimum 28 day compressive strength (f ’c) of 5000 psi.

For these situations the designer shall work with the office of Materials to develop a special provision for the project to address the concrete mix design. In addition, the designer shall place the following note under the “DESIGN STRESSES” heading on the notes and quantities sheet and under the “PIER NOTES” on the pier detail sheet. A similar note shall be placed in the “estimate reference information” under the “Structural Concrete (Bridge)” bid item which addresses the quantity of the higher strength concrete.

All pier concrete shall meet a 28 day design strength of f'c = 5000 psi (35 MPa).

Check with your section leader for approval to use the higher strength concrete.

Methods Memo No. 64: Removal of Corrosion Inhibitor Option on Columns
18 June 2002

In the past, the office has allowed the option of substituting non-coated bars with a corrosion inhibitor admixture for the structural concrete for situations where the column vertical reinforcing, hoops, and footing to column reinforcing were epoxy coated. See bridge substructure notes E713 and M713. However, the Materials Office has expressed concern about the interaction of the corrosion inhibitor with other additives and the aggregate in concrete mixes. In addition, there have been some questions about the overall effectiveness of inhibitors in preventing corrosion. Therefore, until more research is available this option shall no longer be allowed on the plans.

C6.6.4.1.1. Pier cap

C6.6.4.1.1.1. Analysis and design

Methods Memo No. 8: Pier Cap Design Shear Stirrup Spacing (Frame pier cap height has been modified in the manual.)
9 April 2001

When designing a pier cap, try to limit the cap height proportions to the overall dimensions of the substructure. As a general rule, limit the cap height to:

1. 6.0 to 7.5 ft (1800 to 2300 mm) for T-Pier caps.
2. 3.5 to 4.5 ft (1100 to 1400 mm) for frame pier caps. **(Maximum height = 5.5 feet (1.680 m), 4 June 2008.)**

For longer span prestressed bridges, where design of shear steel in the cap may be difficult because of high beam reactions and the cap height becomes excessive, the following options should be considered:

1. Substitute no. 6 (no. 19) bars for no. 5 (no. 15) bars to increase the shear reinforcement area.

2. If additional shear reinforcing is needed, check with the section leader to see if single hooked shear bars can be used (Do not use triple shear stirrups in the caps due to constructability concerns).
3. For T-Pier designs, consider widening the column to eliminate the interior beam reaction on the cantilever.
4. Consider widening the cap to increase the shear capacity of the concrete.
5. For T-Pier designs, consider using Load Factor Design for the cantilever design.
6. For frame piers, consider using load factor for the cap design, however the current in-house program does not give factored loads, so a separate analysis is required.
7. Consider a higher strength concrete for the substructure following the guidelines in MM No. 16 (Use of Higher Strength Concrete).

Check with your section leader for approval.

Methods Memo No. 16: Use of Higher Strength Concrete
21 March 2001

See C6.6.4.1.

Methods Memo No. 6: Pier Cap Design, Shear Stirrup Spacing
9 April 2001

There has been some confusion in the office about what to use for maximum shear stirrup spacing in Pier Caps. Currently when calculating the maximum shear spacing, AASHTO specifications 8.15.5.3.8 and 8.19.3 provide a maximum spacing of d/2, 24 inches (600 mm) or d/4, 12 inches (300 mm) based on the concrete shear stress.

Current specifications on temperature and shrinkage (AASHTO 8.20) limit the maximum reinforcement to no.4 at 18 inches (no. 15 at 450 mm). The current AASHTO LRFD spec. 5.10.8 limits the maximum temperature and shrinkage spacing to 18 inches (450 mm) for any exposed surface with components less than 48 inches (1200 mm) thick.

Based on this information for temperature and shrinkage steel and discussions with the policy group, limit the maximum spacing to 18 inches (450 mm) for shear stirrups.

Methods Memo No. 7: Using hooks near cap ends for development of flexural reinforcing steel
19 January 2001

The office would like to avoid the use of hooks in the flexural reinforcing steel of pier caps. However, if situations come up where you would have to extend the cap to a length that is wider than the slab above, then hooked bars can be used to limit the cap length.

Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009

See C6.6.4.1.3.1.

C6.6.4.1.1.2. Detailing

Methods Memo No. 107: Integral Abutment and Pier Cap Detailing
6 June 2005

See C6.5.4.1.2.

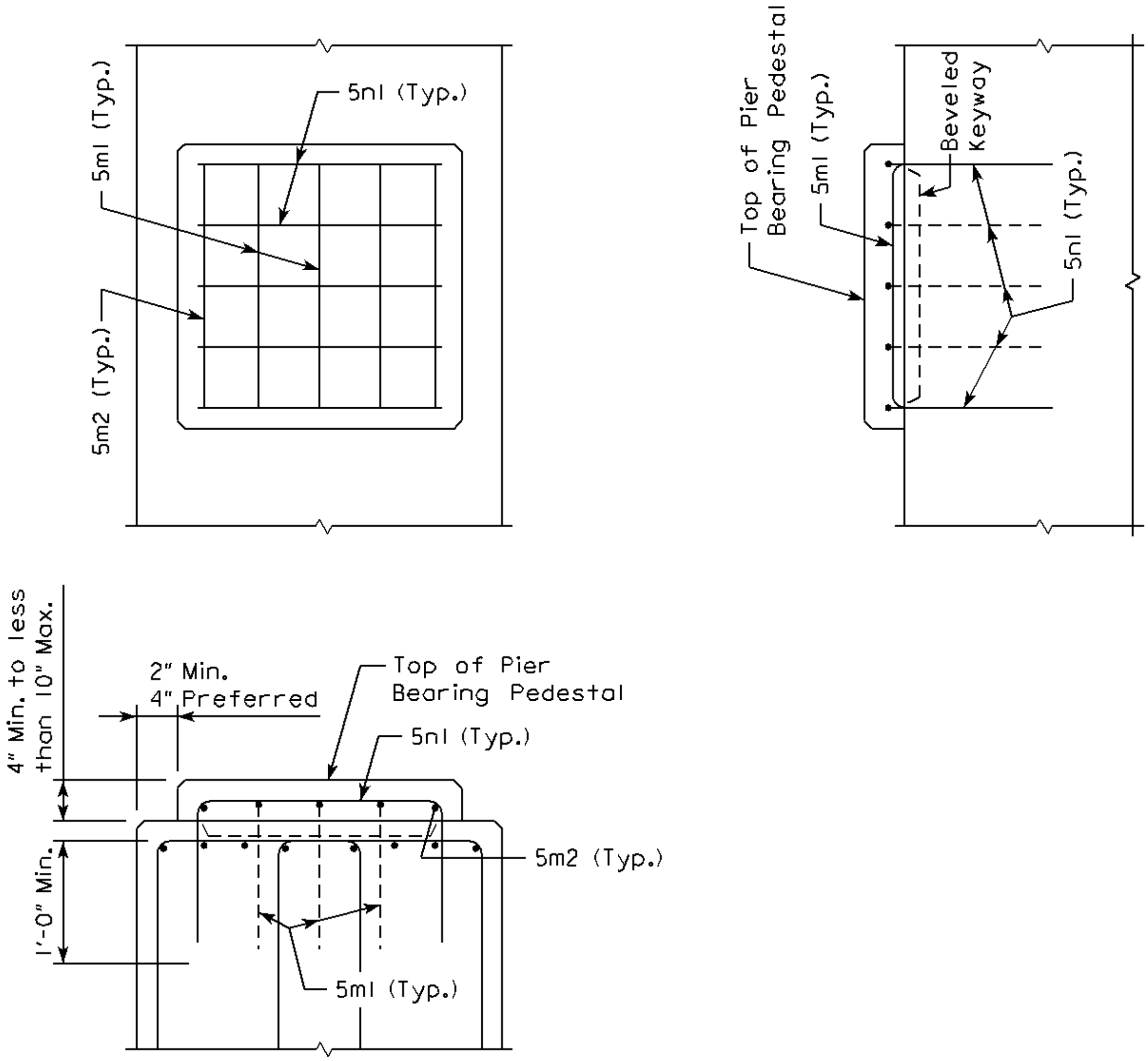
Methods Memo No. 196: Pier Pedestal Guidelines for Aesthetic Piers (LRFD Bridge Design manual 6.6.4.1.1.2)
1 January 2008

With the increased use of aesthetic bridge piers, our office has been using more level non-stepped pier caps with individual concrete pedestals to support the beam bearings. To help provide consistent design and details for the pedestal bearing supports when aesthetic piers are used, the following design guidelines have been adopted.

1. Minimum pedestal height at low step shall be 4 inches.
2. For pedestals from 4 inches to less than 10 inches, use 5m1 and 5n1 bars. (See Attachment A.)
3. For pedestals from 10 inches to less than 18 inches provide a single tie along with the 5m1 and 5n1 bars. (See Attachment B.)
4. For pedestals from 18 inches to 24 inches provide two ties along with the 5m1 and 5n1 bars. (See Attachment C.)
5. Maximum pedestal height shall be 24 inches without special approval from the Section Leader.
6. A horizontal construction joint and keyway are required between the pier cap and pedestal.
7. Minimum bearing edge distance should meet the same guidelines for pier caps {LRFD BDM Figure 6.6.4.1.1.1-1}.
8. The preferred set back from the pier cap vertical face is 4 inches with a minimum set back of 2 inches.
9. If possible the pedestals shall be square, but rectangular shapes may be required for skewed piers.
10. The vertical pedestal bars shall be embedded into the cap a minimum of 12 inches past the main cap reinforcing steel and shall be tied and in place prior to the cap concrete placement.
11. For fixed bearings the anchor bolts should extend into the cap concrete a minimum of 1'-0 for 1¼ inch diameter anchor bolts and 1'-3 for 1½ inch diameter anchor bolts.
12. For expansion bearings, the anchor bolts do not need to extend into the pier cap and may end in the pedestal concrete; however if guides are used to prevent lateral movement, the anchor bolts for the guides shall meet the same embedment requirements as the fixed bearings in No. 11 above.

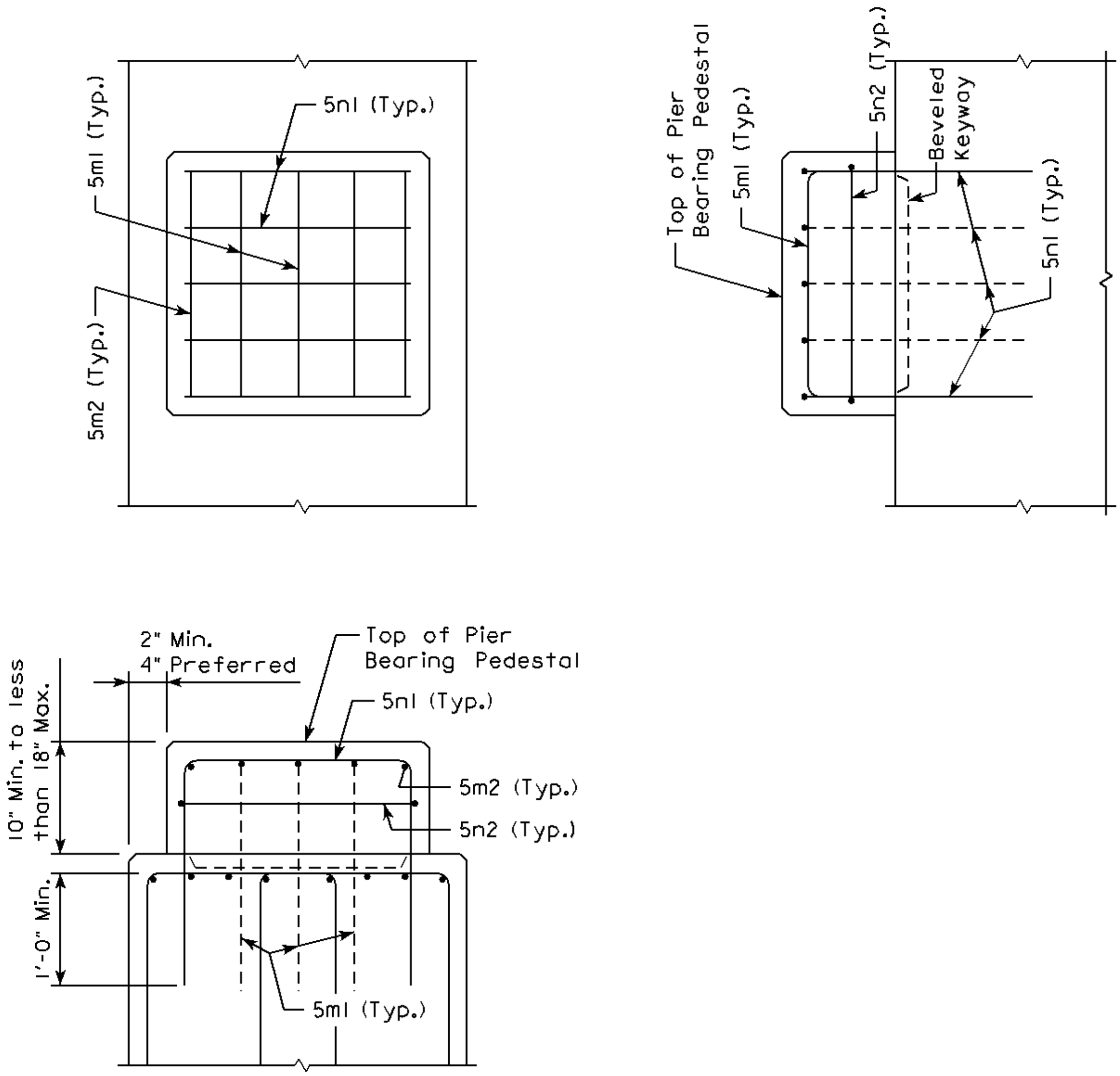
If you have any questions, please check with me.

Attachment A



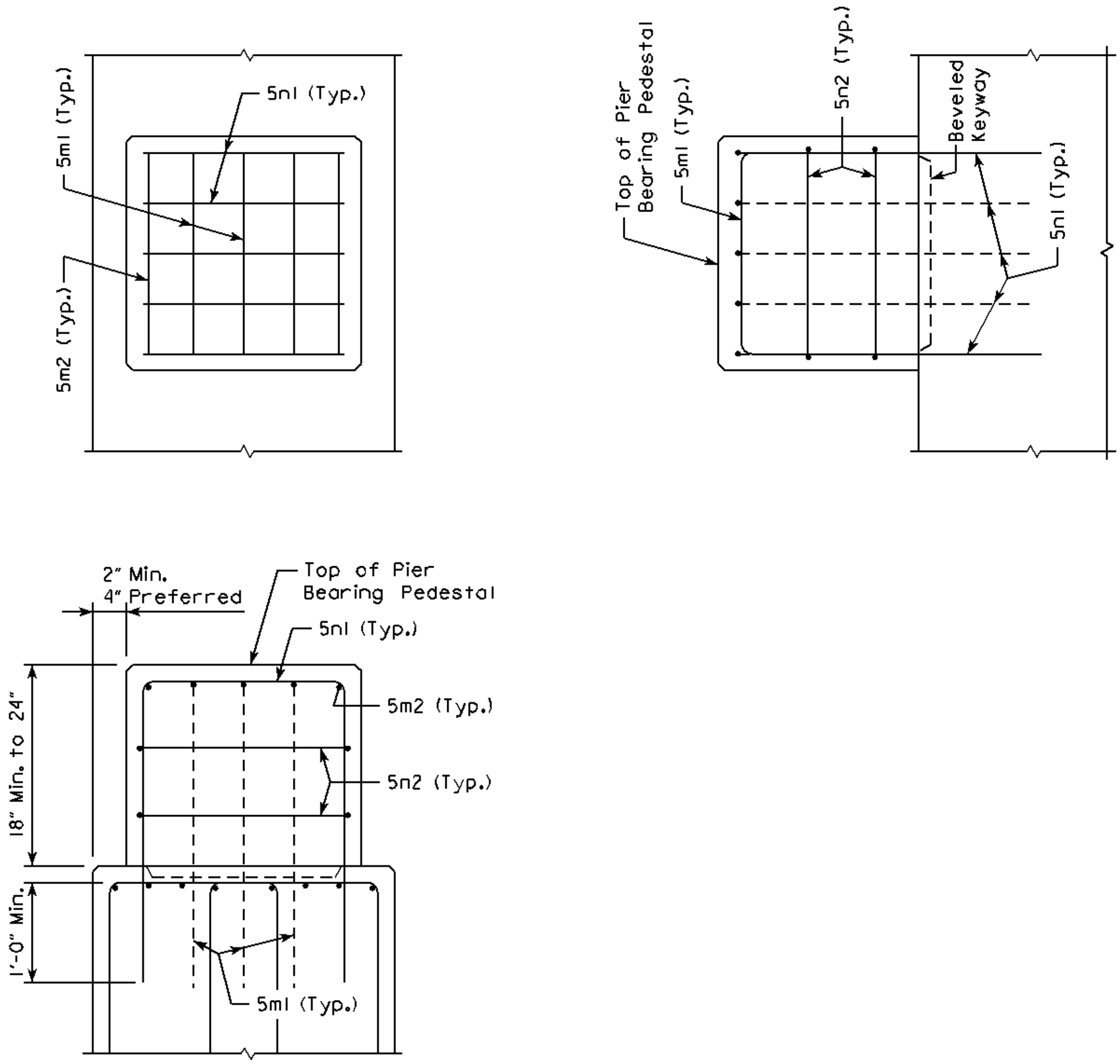
PIER BEARING PEDESTAL

Attachment B



PIER BEARING PEDESTAL

Attachment C



PIER BEARING PEDESTAL

C6.6.4.1.2. Pier column

C6.6.4.1.2.1. Analysis and design

Methods Memo No. 107: Integral Abutment and Pier Cap Detailing
6 June 2005

See C6.5.4.1.2.

Methods Memo No. 5: Maximum T-Pier Heights Based on Kl/r
24 January 2001 (Much of this memo is obsolete, however the options still are valid.)

Questions have been brought up about the maximum column heights that can be used for T-pier design. Based on AASHTO Specification 8.16.5.2.6 the maximum KL/r that can be used for column design without having to do a second order analysis is 100. Under the 100 limit AASHTO allows the designer to use moment magnifiers to take into account slenderness. We would like to limit the height of T-pier columns so that the KL/r ratio is kept below the limit of 100. The mainframe program automatically calculates and uses the moment magnifiers, but is limited to designs below the 100 limit.
If you exceed the 100 limit in the program, a message will come up stating:

> "KL/R>100, THE FOLLOWING P & M ON COL DO NOT INCLUDE SLENDERNESS EFFECTS, DESIGNER SHALL ANALYZE THE STRUCTURE ACCORDING TO AASHTO SECTION 8.16.5.1."

Attached are some hand calculations that were done to give the designers an idea of maximum heights that can be used. Working backwards from the KL/r limit of 100 for a 2.5 ft thick rectangular column, the maximum design height is approximately 36 feet. These calculations are based on using a K of 2.0.

Options that can be considered for increasing the heights of columns are:

1. Increasing the column width to reduce the KL/r to below 100.
2. Substituting round columns for the design.
3. Using a combination of a thicker pier wall with the larger columns placed on top of the wall.

If this situation comes up, check with your section leader for approval on which option to use.

Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009

See C6.6.4.1.3.1.

C6.6.4.1.2.2. Detailing

Methods Memo No. 64: Removal of Corrosion Inhibitor Option on Columns
18 June 2002 (Supersedes Methods Memo No. 42, which has been moved to the appendix for this commentary section)

See C6.6.4.1.

Methods Memo No. 75: End Bar Clearances for Horizontal Construction Joints
6 July 2005

See C6.4.5.

Methods Memo No. 204: General Note on Keyway Dimensions
1 October 2008

See C11.5.2

Methods Memo No. 69: Epoxy Coated Spirals
10 August 2004 (Portion in curly brackets {} regarding lap length superseded by manual text)

It has been brought to our attention, that companies that provide epoxy coating for reinforcing steel can epoxy coat spiral reinforcement and ties in any size. Therefore, in situations where the column reinforcement is required to be epoxy coated (Article 6.6.4.1.2.2, of the Bridge Design Manual), the plans shall show epoxy-coated spirals along with the option of using epoxy coated tied reinforcing [minimum size no. 4 (15)]. {Also, the minimum lap shall be increased by 15 percent to 30 inches (1.15*26 in.) from the current minimum for non-coated reinforcing steel.

The lap length for the epoxy-coated spirals shall be 1 ½ turns or a minimum of 30 inches (760 mm).

The lap length for epoxy-coated ties shall be a minimum of 30 inches (760 mm).}

Methods Memo No. 109: Reinforcement Placement in Round Columns
3 March 2006

In a recent meeting, it was noted that contractors on certain projects were having trouble placing concrete for columns. The designer had used 90-degree hooks to develop the column reinforcement into the cap. The bends were placed on the inside of the columns and restricted the opening for the tremie. Sometimes hooks are required because of increased development length for epoxy-coated steel. However, attempts should be made to limit their use by taking advantage of the reduced development length allowed in article 6.6.4.1.2.2 of the Bridge Design Manual.

If the hook is still required for design, a minimum opening of 12 inches shall be provided in the center of the column for concrete placement with a tremie. A 180-degree hook should also be considered for increasing the opening size.

This change shall be incorporated into all plans currently under development.

C6.6.4.1.3. Pier footing

C6.6.4.1.3.1. Analysis and design

Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009 (Item 4. was edited for clarity 24 November 2009.)

Questions have been raised about the office's policy in regard to mass concrete and when should concrete casting include monitoring of concrete placement and curing temperatures. In addition, at what thickness should reinforcement be provided for temperature and shrinkage along the sides of concrete footings? AASHTO LRFD 5.10.8 requires temperature and shrinkage reinforcing be provided when the footing thickness is greater than 3 ft.

However, based on the Iowa's experience with larger reinforced concrete members, the designers shall use the following guidelines:

1. Concrete footings that are greater than 5 ft in depth should be considered mass concrete (See MM No. 192) and guidelines for controlling and monitoring of the concrete mix temperatures should be included in the plans or specifications.

2. Concrete members other than footings shall be considered mass concrete when the least thickness of the section is greater than 4.0 ft and guidelines for controlling and monitoring of the concrete mix temperatures should be included in the plans or specifications. Typically these requirements are used on larger projects such as border bridges or the Iowa River Bridge on Highway 20.

3. Concrete footings greater than 5 ft in depth shall also include side reinforcing for temperature and shrinkage steel per AASHTO LRFD 5.10.8.

4. All other concrete members shall meet AASHTO LRFD 5.10.8 for temperature and shrinkage steel **with the exception of concrete footings with a thickness of 5 ft or less**.

This policy should be followed on new projects where the design work has not been started. If you have any questions check with me.

Methods Memo No. 192: LRFD Office Guidelines for Temperature and Shrinkage Reinforcing in Pier Footings
1 March 2008

Questions about the use of temperature and shrinkage steel on deep concrete members (specifically footings) when designed by the AASHTO LRFD specifications have been raised recently. The questions were in regard to the need for surface reinforcing on the sides of pier footings and if this reinforcing should be provided. The key issue is at what thickness should footings be defined as “mass concrete”.

Based on our experience with thicker footings, footings over 5 ft shall be defined as “mass concrete”. This continues a policy that was started with Methods Memo No. 3. Therefore, continue the Office’s policy of using reinforcing in the top of the footings for thickness of 5 foot or less where uplift is a problem. For large footings that are greater than 5 ft thick, check with your section leader or me to see if temperature and shrinkage requirements need to be met along the sides and top of the footing and if additional mass concrete notes need to be included in the plans.

Methods Memo No. 3: Punching Shear and Wide Beam Shear
21 March 2001 (Revised 29 January 2003)

There have been some discussions on the design procedures for checking the punching shear and wide beam shear in T-pier footings. The concern involved what should be used for pile loads and allowable concrete stresses. These issues have become more of a problem because of the longer prestressed beams that the office is using and the resulting heavier loads. Designers have had some problems with excessive footing depths (> 5 ft.).

To help clarify the design, the following procedure should be used.

When designing for punching shear:

1. Base the shear stress for punching shear on the pile load P/N (where P is the total axial load and N the number of piles) with bending moments ignored. Be sure to include any allowable overstresses for Group loadings.
2. Use section 8.15.5.6.3 ($v_c = (0.8 + 2/\beta_c)*f'c^{1/2} \leq 1.8*f'c^{1/2}$) of the AASHTO Bridge Specifications for calculating the allowable concrete stresses for punching shear.

When designing for wide beam shear the footing shear stress (v_c) shall be based on the pile design bearing capacity, not the maximum pile load.

The office would like to limit the footing depth to 5 ft. or less, because of the problems caused by heat of hydration can cause in larger concrete sections. Surface cracks can develop due to the temperature differential between the center of the concrete section and the exterior surfaces. Because of the possibility for cracking, special placement may be required for these deeper footings.

If situations develop where the footing depth is excessive (>5ft), check with your section leader for approval.

Methods Memo No. 145 gave rules for checking piles at design and check scour conditions under the AASHTO Standard Specifications. The rules were based on service load design (SLD) and have been updated for the AASHTO LRFD Specifications. The following is a brief summary of the LRFD specifications regarding scour.

Service limit state

- 10.5.2: Consider foundation movements, including movement at the design scour condition.
- 10.5.5.1: Use a resistance factor of 1.0 for deflection at the design scour condition.
- 10.7.2.1: Evaluate overall stability for loss of support due to scour.

Strength limit state

- 10.5.3.1: Consider loss of support due to scour at the design flood.
- 10.5.5.2.1: Factored foundation resistance after design flood scour must be greater than factored load with scoured soil removed.
- 10.5.5.2.3: Use resistance factors specified for geotechnical resistance, structural resistance, and drivability analysis.
- 10.7.3.6: Select pile penetration to be adequate after scour. Consider debris loads during flood event.

Extreme event limit state

- C10.5.4: Design for check flood scour.
- 10.5.5.3.2: Nominal resistance ($\varphi = 1.0$) after check flood scour is to be adequate for unfactored strength limit state loads. For uplift take $\varphi = 0.8$ or less. Consider debris loads during the check flood.
- 10.7.4: Use check flood and resistance factors from 10.5.5.3.2.

For geotechnical design the procedures in Methods Memo No. 145 are superseded by the AASHTO LRFD Specifications, which are more liberal for check scour. After severe flooding, scour will be evaluated by Iowa DOT bridge inspection teams under the Scour Watch program, and therefore, under check scour conditions, safety will be assured by design, field inspection, and bridge closures.

For structural design, Structural Resistance Level - 2 (analogous to 9 ksi) at KL/r of 80 for design scour in Methods Memo No. 145 has been taken as the basic condition and extrapolated under LRFD to Structural Resistance Level – 3. For check scour the maximum slenderness has been taken at 120 (the maximum for a main compression member) for all Structural Resistance Levels. This will result in a small apparent margin of safety for piles under check scour, more than required by the AASHTO LRFD Specifications, which will allow some capacity for bending of the pile under stream flow pressure. If the superstructure is partially or fully inundated by the 100-year flood the designer will be required to check the 500-year flood condition at the extreme event limit state as discussed in BDM 6.6.2.7.

For Table 6.6.4.1.3.1-1 the maximum slenderness ratios for design scour were determined to provide an apparent margin of safety, γ/φ (load factor/resistance factor), of about 3.4 for a compression load. The maximum slenderness ratio of 120 [AASHTO-LRFD 6.9.3] at the check scour condition results in an apparent margin of safety of about 1.4, minimum, for compression load. Because the basic column stability formula was changed from the AASHTO Standard Specifications to the AASHTO LRFD Specifications, the apparent margins of safety in the two specifications cannot be compared directly.

References

Galambos, T.V., Ed. (1998). *Guide to Stability Design Criteria for Metal Structures, Fifth Edition,* John Wiley & Sons, Inc., New York, NY.

Tide, R.H.R. (2001). “A technical note: derivation of the LRFD column design equations,” *Engineering Journal,* third quarter, 137-139.

Example: Scour analysis and design, pier pile, LRFD

Given: Pier pile, Grade 50, HP 10x57
Unfactored axial compression load at the strength limit state, P = 135 kips
Factored axial compression load at the strength limit state, P_u = 210 kips at SRL-2
(This factored load is less than the maximum factored load that would be permissible at SRL-2, (0.6)(365) = 219 kips [BDM Table 6.2.6.1-1].)
Elevation, bottom of footing: 640 feet
Elevation, design scour: 633 feet

Water velocity, design flood: 5 feet/sec
Elevation, check scour: 630 feet
Soil profile: 10 feet silty sand, N = 8
20 feet firm glacial clay, N = 11
60 feet very firm sandy glacial clay, N = 25

Determine pile length considering geotechnical resistance with design scour [BDM 6.2.7]. This condition will control rather than the condition without scour. Also, determine driving resistance in scourable material.

Required geotechnical resistance = P_u/φ_c = 210/0.725 = 289.7 kips

Cutoff after driving	1 foot		
Pier embedment	1 foot		
Silty sand	10 feet (less scour)	(3)(1.2) =	3.6 kips
Firm glacial clay	20 feet	(20)(2.8) =	56.0 kips
End bearing in very firm glacial clay		(16.8)(2) =	33.6 kips
Very firm glacial clay	50 feet	(50)(4.0) =	200.0 kips
Total	82 feet, round to <u>85 feet</u>		293.2 kips > 289.7 kips, OK

Theoretical driving resistance in scoured material

Silty sand	7 feet	(7)(1.2) =	8.4 kips

Check geotechnical resistance with check scour [BDM 6.2.7]

Required geotechnical resistance = P/φ_c = 135/1.00 = 135.0 kips

Cutoff after driving	1 foot		
Pier embedment	1 foot		
Silty sand	10 feet (less scour)	(0)(1.2) =	0.0 kips
Firm glacial clay	20 feet	(20)(2.8) =	56.0 kips
End bearing in very firm glacial clay		(16.8)(2) =	33.6 kips
Very firm glacial clay	50 feet	(50)(4.0) =	200.0 kips
Total			289.6 kips > 135.0 kips, OK

Check structural resistance at design scour [BDM Table 6.6.4.1.3.1-2]

Pile length = (640-633) + 4 = 11 feet < 16.0 feet, OK

Check structural resistance at check scour [BDM Table 6.6.4.1.3.1-2]

Pile length = (640-630) + 4 = 14 feet < 24.1 feet, OK

Determine driving resistance values to be included in plan note.

Dead and live load	= (75)(210/219) =	71.9 tons
Scourable material	= (71.9)(8.4/289.7) =	2.1 tons
Total		74.0 tons

CADD Note E834 on plans (assuming that the pier pile in this example was the controlling pile):

PIER PILES ARE DESIGNED TO ACCOMMODATE THE ABSENCE OF SCOURABLE SOILS ABOVE THE 100 YEAR SCOUR ELEVATION SHOWN IN THESE PLANS. PILES SHALL BE DRIVEN TO 74.0 TONS BASED ON THEORETICAL DRIVING RESISTANCE. THIS INCLUDES 2.1 TONS OF RESISTANCE IN THE SCOURABLE LAYERS, AND 71.9 TONS RESISTANCE FOR DEAD AND LIVE LOAD BEARING CAPACITY.

Methods Memo No. 1: Footing Elevation for Scour and Allowable Unsupported Pile Lengths (The unsupported lengths have been superseded by the manual text. 19 June 2008)
23 March 2001

When calculating the T-pier footing elevations, the following guidelines should be used in the design.

1. Set the bottom of the footing elevation to 6 ft (1800 mm) below the channel elevation.
2. Check the unsupported pile length for the piling assuming the pile length is from the bottom of the footing to 4 ft (1200 mm) below the scour elevation. Based on the unsupported length, select the following steel pile size.

Pile Size	Unsupported Length
HP10x42 (HP 250x62)	14 ft. (4200 mm)
HP 12x53 (HP 310x79)	17 ft. (5100 mm)
HP 14x89 (HP 360x132)	20 ft. (6000 mm)

Note: The preferred pile size to use is HP10x42 (HP 250x62) piles if possible. Consider lowering the bottom of the footing if only an additional 1 or 2 ft. (300-600 mm) is required to meet the 14-ft. (4200-mm) limit for HP10x42 piles (HP 250x62).

3. To provide additional stability for the pile group the piles on the outside rows should be battered at 4:1 slopes.

If other conditions are encountered where these guidelines cannot be followed, then check with your section leader for additional details.

Commentary:

These lengths were based on the following office policies:

a. For the HP10x42 (HP 250x62) pile, the unsupported pile length is 10 ft (3000 mm) of exposed pile plus 4 ft (1200 mm) below the scour line for a total length of 14-ft (4200-mm).
b. Pin connections for the piling are assumed at the bottom of the footing and at the point 4 ft. (1200-mm) below the scour line. Using a K of 1.0, the KL/r of an HP10x42 (HP 250x62) is approximately 70.
c. Using the KL/r of 70 the unsupported lengths for the HP 12x53(HP 310x79) and HP 14x89(HP 360x132) were calculated.
d. No impact on the footing piles was assumed in the calculations for the scour condition.
e. These lengths give a factor of safety of 3.38 for a 55-ton (490 kN) bearing pile (9 ksi, 60 MPa) and 5.08 for a 37-ton (330 kN) friction pile (6 ksi, 40 MPa).

The office practice of using 6 ft for the depth of the footing below streambed is based on a past AASHTO Specification (1983 spec 4.4.2) that is still followed by the office. The guidelines for the KL/r are conservative when compared with the AASHTO Specification Table 10.32.1.A for concentrically loaded columns. The office is conservative on this issue because:

1. Pile loads will not be perfectly concentric in a footing.
2. Lateral loads will be applied to the piles during scour conditions.

3. Any damage that may happen to the piles during scour conditions or loss of section because of corrosion cannot be inspected later.

Methods Memo No. 145: Pier Foundation Design and Check for Scour Conditions
4 September 2007

See the Appendix for obsolete and superseded memos in this commentary.

Methods Memo No. 117: Pile Cutoff for Battered Piles
20 July 2005

See C6.2.5.

Methods Memo No. 9: Battered Pile Capacity and Lateral Load Capacity for Pier Design
9 April 2001

See C6.2.4.

Methods Memo No. 2: Top Bar Factor for Footing Reinforcing Steel Development Length
2 January 2001

There have been questions on whether the office should be using a 1.4 top bar factor for the development length check of the bottom mat of reinforcing steel for pier footings when the mat is placed above the piling. Based on the AASHTO Specification 8.25.2.1 the 1.4 factor should be used any time there is more than 12 inches of concrete placed below the reinforcing. After discussions with the section leaders, it was decided to use the 1.4 factor for the bottom mat of steel when it is placed above the piles. If you have problems getting the development length to work, the following options can be considered:

1. Try and use a smaller size bar to decrease the development length.
2. Reduce the development length by using AASHTO Specifications 8.25.3.2 where the development length can be decreased by a factor based on the Areas Steel Required / Area Steel Provided.
3. Drop the mat of steel so that it is 6 inches clear from the bottom of the footing. If you have a large number of piles in your footing, this option may not be your best choice. During driving the piles may shift from their correct location making it difficult for the contractor to place reinforcing around the piles.
4. If using options 1 through 3 and you still cannot meet the development length, then provide a hook at the bar ends. Check with your section leader for approval.

C6.6.4.1.3.2. Detailing

Methods Memo No. 50: Detailing of Footing to Column Rebar
28 September 2001

When detailing the footing to column bars, limit the length of rebar extending above the footing to a maximum of 15 ft. (4500 mm).

Recently there was a project (Hardin 499) where the reinforcing steel was detailed to extend approximately 30 ft. (9144 mm) above the footing. This requires contractors to work with bar lengths that are not reasonably constructible. In this particular project the reinforcing steel length was changed to include a 10.5 ft. (3200 mm) lap.

Please include this revision in all projects that are currently being developed.

C6.6.4.1.4. Seal coat

Methods Memo No. 21: Use of "Excavate and Dewater" Bid Item
5 February 2002

There has been some confusion on the use of the bid item "Excavate and Dewater". The following issues have been raised in regard to the item.

- When is the bid item "Excavate and Dewater" used?
- Who determines when to use the bid item?
- Is the "Excavation, Class 21" bid item required also?
- What design criteria are used for seal coats?

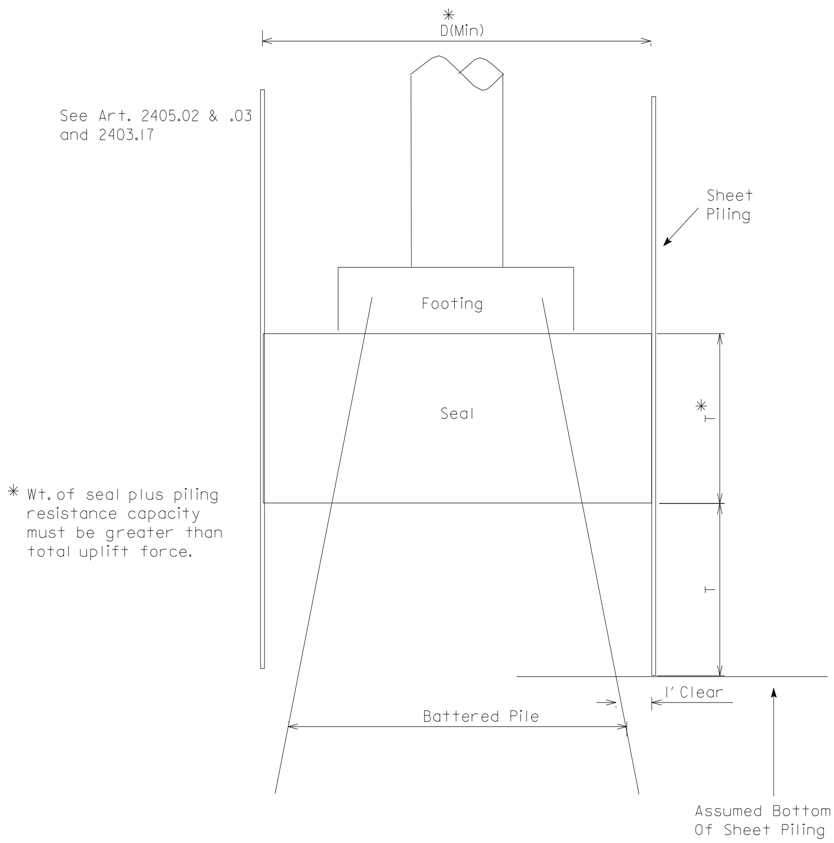
A. <u>When is the bid item "Excavate and Dewater" used?</u>

The "Excavate and Dewater" bid item was developed for use in the construction of piers for the large Iowa Rivers where the soil, at and below the footing excavation, is predominantly sand. When a footing is located below the water table in these soil conditions, the bottom of the sheet pile cofferdam may be difficult to seal or the length of sheet pile required to seal the cofferdam would not be economical. A concrete seal coat (See Detail A) can be used in these situations. By using the seal coat, it allows the construction of the footing in the dry. A seal coat may also be required in situations where underlying soft rock may be porous.

General procedure for installation of a seal coat is as follows:

1. Drive sheet piles to form cofferdam.
2. Drive H-piles for bridge footing.
3. Place concrete seal coat.
4. Pump cofferdam dry.
5. Place bridge footing in the dry.

Anytime a cofferdam is required to construct a footing and the permeability of the soil is in question, the design engineer should check with their section leader and consult with Design's Soils Section.



Detail A

B. Who determines when to use the bid item?

If the permeability of the soil is in question, the design engineer should check with his/her section leader and the Soils Section to see if “Excavate and Dewater” is required. This bid item should generally not be used for bridges over small or medium sized streams.

C. Is the “Excavation, Class 21” bid item required also.

When “Excavate and Dewater” is used, the Standard Specification 2405.14 states that both Class 20 and 21 along with the cost of cofferdams, seal coats, pumps and all other equipment required are included. Therefore, the “Class 20” bid item, “Class 21” bid item and the ”Excavation Classification Line” are not required.

D. What design criteria are used for seal coats?

Design guidelines for the seal coat are as follows:

1. Design bond between the seal coat and pile = 5 psi (The lower allowable bond is due to the uncertainty of the bond strength of steel to concrete when the placement is in water.
2. Include the weight of the seal coat in the design of the piling.
3. When designing the size of the cofferdam, assume the tip of the sheet pile cofferdam extends 1 ft from the edge of the piling (See Detail “A”).
4. Base the elevation of the top of the cofferdam and the thickness of the seal coat on a Q_{25} flow.

5. Base the design on the weight of the seal coat and bond strength between the piling and seal coat to counteract the water pressure at the bottom of the seal coat. Assume a factor of safety of 1.1.
6. Because of the uncertainty of whether the seal coat is going to be used, the pier needs to be designed for both situations (with or without the seal coat).

When using the "Excavate and Dewater" bid item, seal coat details shall be provided on the plans and standard note E832 or M832 shall be placed on the pier sheet.

As described in the Attachment "A" below, the bid item was developed to:

1. Provide a method of bidding that is fair to the contractor.
2. Allow the designer to specify certain conditions (cofferdam size, elevation, and length) to insure the structural integrity of the bridge.
3. Eliminates as much negotiating as possible to reduce the work orders and paper work required by the field to complete the project.

Attachment "A" (Development of "Excavate and Dewater" Bid Item)

The following is some background information on the development of the "Excavate and Dewater" bid item taken from a memo from Bill Lundquist to his design section (12-3-80).

The question arises as to how best to specify and bid the excavation and dewatering necessary to construct a pier in a large inland Iowa river where the soil, at and below the footing excavation, is predominantly sand. It has been our understanding that a seal coat is necessary to allow the pier footing to be placed in the dry. Our practice has been to design the seal thickness, specify it on the plans, and bid it and the excavation necessary to place it as definite bid quantities. If the contractor chooses to try to construct the pier without the seal coat, permission must be obtained from the central office. If he is successful in this construction, a work order results, deleting the cost of a seal coat concrete and the Class 21 Excavation that was saved. However, the contractor is paid an adjusted price for these two items that would reflect the fixed costs he might have included in the cost of the seal and excavation. These fixed costs he might have included cofferdam, overhead, profit, bid bond and insurance, etc. The determining of this adjusted price is a negotiated item and can be hard to agree upon.

The new bridge across the Cedar River at Vinton (878 Benton) is a good example of the above. The pier footings sit in medium sand, about 50 feet deep, overlying firm glacial clay. The footings are about 10 feet below streambed, to protect against scour, and the seal coats that were specified on the plans were designed for a head of the seal coat specified for each. The contractor worked inside a steel sheet pile cofferdam at each pier and was able to pump the cofferdams dry. Seepage was controlled by the use of a sump and a pump. All piles and footings were driven and constructed in the dry. A work order was processed for the elimination of the seal and excavation, and after finally negotiating an adjusted price to cover fixed costs; a savings was realized by the State of Iowa.

Several problem areas arise in specifying and bidding dewatering costs in the above manner. First, of course, is negotiating the adjusted price for the materials deleted. Second, is the possibility of the contractor unbalancing his bid. Bidding the seal coat low and adjusting other items upward on the assumption, he will not place the seal, but will place the adjusted items. His loss in total dollars then is less than if he were to bid the cost of the seal coat more accurately.

It appears that a better way to bid the costs required to dewater is by an all-inclusive lump sum bid "Excavate and Dewater." Included in this would be all costs of labor and materials required to allow the pier footing to be constructed in the dry. This could include cofferdams, concrete seal coats and the excavation required to place the seal coat, well points, earth dikes, or any approved method of dewatering plus all pumping, bailing, or drainage required. The size and thickness of seal coat, if used, would be shown on the plans. The pier would be designed to be stable if the seal coat was not used and the footing piling designed to carry the weight of the seal coat if used.

An example of the above type of bidding is designs 577 and 677 Black Hawk, I-380 over Cedar River. On this project, the contractor again chose to construct the piers without seal coat, again by the use of steel piles, pumping, and a sump and pump. In this instance, though, no work order and negotiation will be required due to the elimination of the seal coats. The burden of bidding, gambling on the necessary use of seal coats, is placed upon the contractor. He is required by specification to do whatever is approved or required by the Engineer to insure that the footings can be placed in the dry.

We want to provide, on the plans, a method of bidding that is fair to the contractor, but yet specifies certain conditions to insure the structural integrity of the bridge. We also want to eliminate as much negotiating as possible to reduce the work orders and paper work required to complete a project. It seems that bidding by the "Excavate and Dewater" method would best satisfy these criteria.

The designer should review carefully the situation at hand before deciding to follow this bidding method. There may be instances where bidding in the present manner than by "Excavate and Dewater" is the best method to follow. Also, the usual bridge over a small or medium sized stream should not require any special mention of dewatering or means to do so.

C6.6.4.1.4.1. Analysis and design

For this LRFD manual the loads, resistances, and factors were calibrated directly from service load design (SLD) in Methods Memo No. 154. Computations and checks under either SLD or LRFD should give the same results.

Methods Memo No. 21: Use of "Excavate and Dewater" Bid Item
5 February 2002

See C6.6.4.1.4.

Methods Memo No. 154: Design of Cofferdam Seal Coat
17 November 2006

The present office policy regarding design of a seal coat allows a design bond stress of 5 psi between a steel H-pile and the seal coat concrete [Methods Memo No. 21]. The allowable bond stress generally is based on the minimum of the policies in other states. The 13th edition of the AASHTO standard specifications permitted a higher value of 10 psi, but this higher value was removed from the specifications in later editions.

Because its value of 5 psi was not based on tests, the Florida DOT recently funded research to determine a more realistic value. On the basis of the research, the Florida DOT increased the allowable to 36 psi for all contact area between steel piles and seal coat concrete [Florida DOT 2000]. Researchers recommended a higher value, 145 psi, but over an area limited by the size of the pile [Mullins et al 2002].

Revising the present office policy to increase the allowable bond stress would have the advantage of reducing the thickness of seal coat for major river bridges and, in some cases, reducing the number of extra piles required to carry uplift on the seal coat. Therefore, the office policy is changed as follows.

- The allowable bond stress between a steel H-pile and seal coat concrete shall be 10 psi.
- The total uplift load carried by each pile also shall be limited to the friction resistance available for the pile embedment in the soil. Friction resistance shall be determined from the latest edition of the Soils Design Section's "Foundation Soils Information Chart, Pile Foundation."
- The minimum seal coat thickness shall be 3 feet (1.000 m).
- The maximum seal coat thickness for typical river conditions should be limited to 6 feet (2.000 m). Greater depths may be used with approval by the section leader.

- The minimum factor of safety shall be 1.1 for the following combination of vertical forces: seal coat weight, hydraulic head and allowable pile uplift load for all piles considering the smaller of the H-pile/seal coat bond or H-pile/soil friction force. Bond and friction forces include safety factors, so the overall factor of safety will be greater than 1.1.

- The maximum allowable flexural tension stress in the seal coat shall be 250 psi. This stress should be checked for uplift on the cantilevered seal coat beyond the pile group and any other condition that causes significant flexural stress. The Iowa DOT standard specifications provide safety by requiring a minimum flexural strength of 500 psi in test beams before the cofferdam can be dewatered [IDOT SS 2405.03].

References:

Florida Department of Transportation. (2000). "Table 5.3." *Structural Design Guidelines*. Florida Department of Transportation, Tallahassee.

Mullins, G., Sosa, R., Sen, R. and Issa, M. (2002) "Seal Slab/Steel Pile Interface Bond from Full-Scale Testing." *ACI Structural Journal,* 99(6), pp 757-763.

C6.6.4.1.4.2. Detailing

C6.6.4.2. Pile bents and diaphragm piers

C6.6.4.2.1. Analysis and design

Methods Memo No. 19: Guidelines for Fully Encased Pile Bents for Reinforced Concrete Slab Bridge (Revised in text of manual 22 December 2008; see bold text below.)
18 April 2001

When designing and detailing fully encased pile bents the following guidelines should be used.

1. The encasement shall have a minimum width of (20") **22"** for (HP10x42) **HP-10** piles, (and) 24" for (HP12x53) **HP-12** piles**, and 26" for HP-14 piles**. These minimum encasement widths will provide enough tolerance between the edge of the pile and the edge of the encasement should the driven piles be out of alignment.
2. The minimum distance from the centerline of the battered outside piles to the end of the encasement shall be 18". This will ensure enough room for placement of the reinforcement in the end of the encasement and provide additional protection for the piling.
3. Fully encased pile bents for continuous concrete slab bridges shall use a pier cap keyed to the bottom of the slab when the movements due to temperature and shrinkage and skew conditions warrant. See memo to Office from Gary Novey (March 29, 1999, "Use of Non-Monolithic Caps on Reinforced Concrete Slab Bridges") for additional information.
4. When non-monolithic caps are used, the ends of the beveled keyway(s) are to be lined with 1" thick strips of preformed joint filler to allow for the movement of the superstructure along the skew.

C6.6.4.2.1.1. Steel H-piles

Steel H-pile, pile bent example:

Given: Three-span (67'-6, 80'-0, 67'-6) PPCB bridge over gully without stream, 218 feet long with 40-foot roadway and 15-degree skew
Steel H-piles

Factored total DC + DW + LL + IM load at pile bent = P_u = 1800 kips
Maximum height to underside of non-monolithic cap = 18 feet
Ground slope along pile bent 1:30 maximum
Soil profile: 0-3 feet: top soil
3-75 feet: firm - very firm glacial clay with N = 14

(1) Determine size and number of H-piles for structural condition above ground

Check applicability of Bridge Design Manual simplified method [BDM 6.6.4.2.1]
Bridge length: 218 feet < 250 feet …OK
Roadway width: 40 feet < 44 feet …OK
Skew: 15 degrees < 45 degrees …OK
Thermal expansion for steel substructure [AASHTO-LRFD 3.4.1, p3-11]:
(80/2)(12)(0.000006)(50)(1.0) = 0.144 inches < 0.45 inches …OK
Ground slope: 1:30 < 1:10 …OK

Choose pile shape [BDM Table 6.6.4.2.1.1]
Try HP 12x53 because H = 18 feet (> 16 feet for HP 10x57)

Determine number of piles [BDM Table 6.6.4.2.1.1]
$n = P_u/\varphi_c P_n$ = 1800/(0.7)(192) = 13.3, try 14 piles
Spacing = 3'- 0 3/4 > 2.50 feet …OK [spacing from H40-06 series sheet H40-48-06]

(2) Check HP 12x53 structural condition in the ground

Per pile, P_n = 224 kips [BDM Table 6.2.6.1-1]
For bent, $P_r = n\varphi_c P_n$ = (14)(0.6)(224) = 1881.6 kips > P_u = 1800 kips …OK

(3) Check geotechnical condition, determine pile length, and write plan note.

Compute required nominal geotechnical resistance per pile.
$P_n = P_u/\varphi_c n$ = 1800/(0.725*14) = 177.3 kips

Determine pile length.

Cutoff	1 foot		
Cap embedment	1.5 feet		
H	18 feet		
Encasement	3 feet		
Firm - very firm glacial clay	27 feet	(27)(3.2)= 86.4	
Subtotal			86.4 kips

Required additional resistance = 177.3-86.4 = 90.9 kips
End bearing is not to be considered [BDM 6.2.7].
Required length in firm -very firm glacial clay = 90.9/4.8 = 18.9+ feet

Firm - very firm glacial clay	19.0 feet	(19.0)(4.8)	= 91.2
Totals	69.5 feet, round to 70 feet		177.6 kips

Check driven length: 27+19 = 46 feet > 21 feet …OK

Compute theoretical driving resistance values to be included in plan note.

Dead, live, and dynamic load = (46)(13.3/14) = 43.7 tons

CADD Note E720 on plans [BDM 11.8.2]: THE DESIGN BEARING FOR THE PIER PILES IS 43.7 TONS.

Use 14 – HP 12x53 piles, 70 feet long.

Because the piles are designed to Structural Resistance Level – 1 (SRL-1) no drivability analysis is required [BDM 6.2.6.1].

C6.6.4.2.1.2. Prestressed concrete piles

C6.6.4.2.1.3. Concrete-filled steel pipe piles

C6.6.4.2.2. Detailing

Appendix for obsolete and superseded memos

Methods Memo No. 1: Footing Elevation for Scour and Allowable Unsupported Pile Lengths 23 March 2001

When calculating the T-pier footing elevations, the following guidelines should be used in the design.

1. Set the bottom of the footing elevation to 6 ft (1800 mm) below the channel elevation.
2. Check the unsupported pile length for the piling assuming the pile length is from the bottom of the footing to 4 ft (1200 mm) below the scour elevation. Based on the unsupported length, select the following steel pile size.

Pile Size	Unsupported Length
HP10x42 (HP 250x62)	14 ft. (4200 mm)
HP 12x53 (HP 310x79)	17 ft. (5100 mm)
HP 14x89 (HP 360x132)	20 ft. (6000 mm)

Note: The preferred pile size to use is HP10x42 (HP 250x62) piles if possible. Consider lowering the bottom of the footing if only an additional 1 or 2 ft. (300-600 mm) is required to meet the 14-ft. (4200-mm) limit for HP10x42 piles (HP 250x62).

3. To provide additional stability for the pile group the piles on the outside rows should be battered at 4:1 slopes.

If other conditions are encountered where these guidelines cannot be followed, then check with your section leader for additional details.

Commentary:

These lengths were based on the following office policies:

a. For the HP10x42 (HP 250x62) pile, the unsupported pile length is 10 ft (3000 mm) of exposed pile plus 4 ft (1200 mm) below the scour line for a total length of 14-ft (4200-mm).
b. Pin connections for the piling are assumed at the bottom of the footing and at the point 4 ft. (1200-mm) below the scour line. Using a K of 1.0, the KL/r of an HP10x42 (HP 250x62) is approximately 70.

c. Using the KL/r of 70 the unsupported lengths for the HP 12x53(HP 310x79) and HP 14x89(HP 360x132) were calculated.
d. No impact on the footing piles was assumed in the calculations for the scour condition.
e. These lengths give a factor of safety of 3.38 for a 55-ton (490 kN) bearing pile (9 ksi, 60 MPa) and 5.08 for a 37-ton (330 kN) friction pile (6 ksi, 40 MPa).

The office practice of using 6 ft for the depth of the footing below streambed is based on a past AASHTO Specification (1983 spec 4.4.2) that is still followed by the office. The guidelines for the KL/r are conservative when compared with the AASHTO Specification Table 10.32.1.A for concentrically loaded columns. The office is conservative on this issue because:

1. Pile loads will not be perfectly concentric in a footing.
2. Lateral loads will be applied to the piles during scour conditions.
3. Any damage that may happen to the piles during scour conditions or loss of section because of corrosion cannot be inspected later.

Methods Memo No. 4: Individual Punching Shear and Beam Shear
19 January 2001

Questions have been brought up about whether punching shear of individual piles need to be checked in the footing design of piers. After reviewing the calculations done by the Methods Section (See attached sheets), the following comments can be made:

1. Corner piles are the only piles that need to be checked individually for punching shear and beam shear.
2. Beam shear controls the design.
3. For piling using 55 tons of bearing a minimum footing thickness of 32 inches is required.
4. For piling using 37 tons of bearing a minimum footing thickness of 28 inches is required.

Based on the analysis, if your footing depth is over 32 inches then individual piling shear does not need to be checked. This analysis was based on a rectangular footing with the center of the corner piles placed 1.5 ft. from the edge of the footing. For special situations, individual punching shear may still have to be checked.

Methods Memo No. 18: Minimum Steel for Column Design
18 April 2001 (Superseded by AASHTO-LRFD 5.7.4.2 provisions for minimum reinforcement)

For design of compression members AASHTO Standard Specification article 8.18.1.2 allows for a reduction of the gross section as follows:

> "when the cross section is larger than that required by consideration of loading, a reduced effective area may be used"

Based on the reduced effective section the designer can then calculate the minimum reinforcing required using one percent of the reduced section.

However, a question has been raised whether there is a minimum area of reinforcing that is required based on the gross area of the section even if a reduced effective area is used. ACI 318 10.8.4 requires that the minimum area of steel not be less than 0.5 percent of the gross area of the concrete section even if a reduced section is used. In addition, the new AASHTO LRFD Specification 5.7.4.2 requires the area of reinforcing be limited to a minimum of 0.7 percent of the gross area.

Based on the new requirements in the LRFD Specification, the office policy will be to limit the minimum area of steel to 0.7 percent of the gross area of the column section.

Methods Memo No. 42: Use of Corrosion Inhibitor Option on Columns

30 August 2001 (Superseded by Methods Memo No. 64 in C6.6.4.1)

When the design load requires the use of higher strength concrete in piers, the option that we currently give the contractor for using a corrosion inhibitor additive in the concrete rather than using epoxy coated reinforcing in the columns shall not be allowed.

There is concern in the Materials Office about the effects of the corrosion inhibitor on the new concrete mixes that are being used to get the higher concrete strengths, so until more research is available this option shall not be allowed.

Methods Memo No. 145: Pier Foundation Design and Check for Scour Conditions
4 September 2007 (Partially supersedes Methods Memo No. 1 and has been superseded by manual changes for LRFD 24 December 2008)

In bridge scour discussions with the Preliminary Design Section, it was brought to my attention that FHWA is requiring geotechnical and structural check of bridge foundations based on scour levels during flood conditions. Therefore, the Preliminary Design Section will begin providing two scour elevations (design and check) on the TS&L sheets for final design to use [BDM 3.2.2.6.1, in process]. The design scour is the based on a 100-year or lesser flood, depending on which results in the more severe condition. The check scour is based on the 500-year or lesser flood depending on, which condition results in the more severe scour.

The analysis that will be required for the two scour conditions are as follows:

1. Pile geotechnical (bearing) capacity under design scour

   The design of the pile geotechnical capacity shall be made neglecting the soil side friction above the design scour elevation. Typical safety factors apply to this analysis.

2. Pile geotechnical (bearing) capacity under check scour

   The check of the pile geotechnical capacity shall be made neglecting the soil side friction above the check scour elevation using a factor of safety for the soil resistance of 1.1.

3. Pile stability check of the unsupported length.

   Piles subject to scour shall be designed and checked for stability. Piles should be designed with a factor of safety of at least 4.0 for the design scour (100-year or lesser flood, depending on which results in the more severe condition).
   In addition, piles should be checked to ensure that the factor of safety is at least 2.2 for the check scour condition caused by the 500-year or lesser flood. The slenderness ratio, KL/r, shall not exceed 120 [AASHTO-I 10.7.1].

   Unless more specific information is available, the unsupported steel H-pile length for a scour condition may be assumed to extend from a simple support at the bottom of the footing to an inflection point 4 feet (1200 mm) below the scour elevation. Preferred pile size is HP 10 x 57 (HP 250 x 85), and the designer may lower the footing as much as 2 feet (600 mm) to permit use of that pile size. Allowable unsupported pile lengths are given in the table below.

   Table of allowable unsupported Grade 50 H-pile lengths

Pile size	Unsupported length for piles with 6 or 9 ksi (41 or 62 MPa) axial stress, feet (m)		Unsupported length for piles with 12 ksi (83 MPa) axial stress, feet (m)	
	Design scour	Check scour	Design scour	Check scour
HP 10 (HP 250)	16 (4.877)	24 (7.315)	6 (1.829)	20 (6.096)

HP 12 (HP 300)	19 (5.791)	28 (8.534)	7 (2.134)	24 (7.315)
HP 14 (HP 350)	23 (7.010)	35 (10.668)	8 (2.438)	30 (9.144)

See Attachment A for background information and summary of calculations.

Attachment A

Grade 50 computations for H-piles loaded to 9 ksi

Methods Memo No. 1 used a FS = 3.38 for 9 ksi loading. Because the allowable axial stress in the pile always is based on KL/r, that slenderness is the governing factor. The MM No. 1 unsupported lengths all are approximately 70 for Grade 36 H-piles.

In AASHTO Specifications Article 10.7.1, the limiting slenderness for main compression members is set at 120. The usual FS = 2.12.

With our upgrade to Grade 50, at a slenderness of 80 with F_a = 9 ksi, the FS = 4.0. This FS is larger than the present FS even though the slenderness is greater. If we set KL/r = 120, the maximum for a main member, with F_a = 9 ksi the FS = 2.2, which is larger than the ordinary FS = 2.12.

It seems reasonable to set the 9-ksi pile limits based on slenderness, 80 for design scour and 120 for maximum scour and to revise the manual.

Appendix for technical documents

See separate document.